Theory and Practical Application of Vacuum Consolidation at the site of Camau Power Plant in Vietnam

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Vacuum Consolidation has been applied in the past in several countries with various successes. China has had the earliest trials due to unavailability of surcharge fill, but the real industrial application by understanding the mechanism, keeping a permanent gas phase allowing to maintain the Vacuum was developed by Professor J.M. Cognon. Theory and application are described in this presentation.
The Vacuum works at Ca Mau Power-Fertilizer are located at the southern tip of Vietnam, near Ca Mau city, and do comprise two Combined cycle power Plant, each with a capacity of 720 MW.
The natural ground is recent alluvial soil consisting of a 17 m thick soft clay of very poor characteristics, overlaying layers of firm and stiff clay, over-consolidated.
The challenge consisted in providing a stable platform at an average elevation of +2.0 m (Hon Dau), above the 100 years flood level, within a very short period (fast track project), in an area poor in backfill materials. The solution of PVD + surcharge was used when compatible with the time allocated and Menard Vacuum was used for the higher load bearing capacity areas (up to 10 t/m²) or when time available was too short.
The Soil Improvement works were completed on time. The monitoring system allowed to validate the end of the consolidation through back analysis, using principally the ASAOKA method and the Pore pressure decrease method.

1. INTRODUCTION

This document presents the first successful application in Vietnam of the Vacuum Consolidation method.
The site is located in the southern tip of Vietnam, in the Ca Mau province over a former green field area. An area of 90 Ha is dedicated to the creation of a fertilizer-power-gaz complex and the first phase consisted in the construction of a 720 MW combined cycle power plant. A second phase was meant to the construction of a second power plant of same capacity and the final phase is for the construction of an 800,000 tonne/year Urea and Ammonia fertilizer plant.

2. PROJECT PRESENTATION

2.1. Foreword.

The challenge of the project was to raise and to stabilise a platform with an average thickness of 2 m on a 17 m soft clay and to improve it further to various load bearing capacity while remaining within residual settlement compatible with raft and pad footing foundation system.
The project was also on a fast track basis.

2.2. Project description.

The Power plant consisted in various technical and non technical buildings around the core of the Power plant, i.e. the Power Block (Two gaz turbines and one steam turbine).

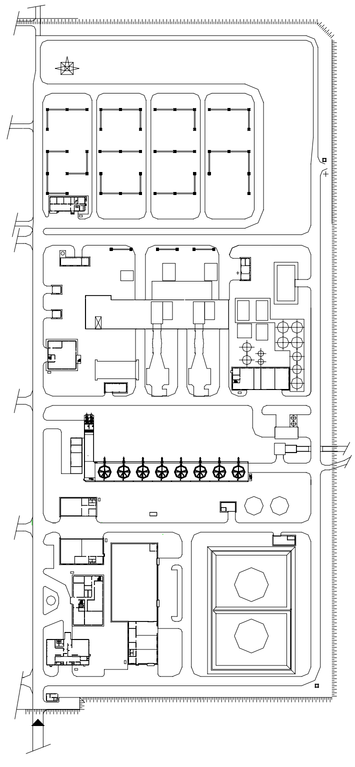

Figure 1 – General Site layout

The design criteria retained for the consolidation were:

- a residual settlement not to exceed 10 cm/10 years in all the built up area, for each and every load bearing capacity considered
- a minimum load bearing capacity of 2t/m² for all the built up area, except
- for the power block, where a bearing capacity of 5t/m² was required,
- and below the cooling towers where a load bearing capacity of 8 t/m² was also required,
- and below some tanks where a load bearing capacity of 10t/m² was needed.

The most critical part of the plant was the delivery of the Power block, to be completed within 8 months from the starting of installation of the first vertical drain.

2.3. Subsoil conditions.

An extensive Soil Investigation campaign was carried out covering the whole of the complex area. The thickness of the soft clay layer was consistent, varying from 16 to 17 m; overlaying a firm to stiff clay layer.

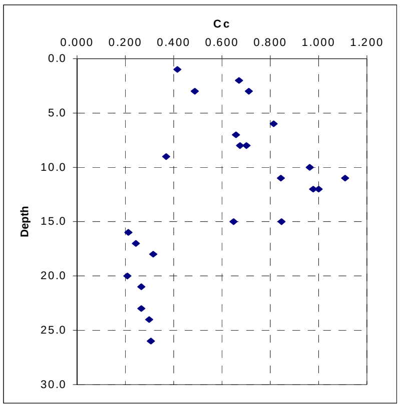

Figure 2 – Cc : Consolidation Coefficient

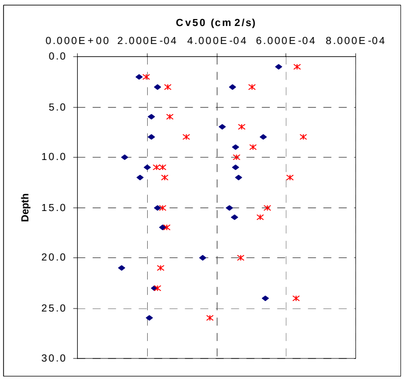

Figure 3 – Cv : Vert. Consolidation Coefficient

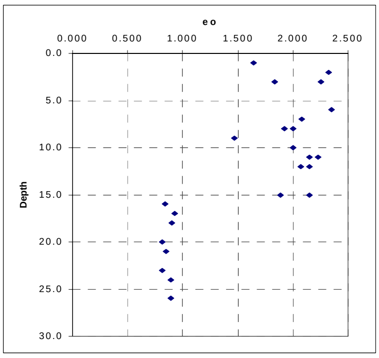

Figure 4 – e_o : Void Ratio

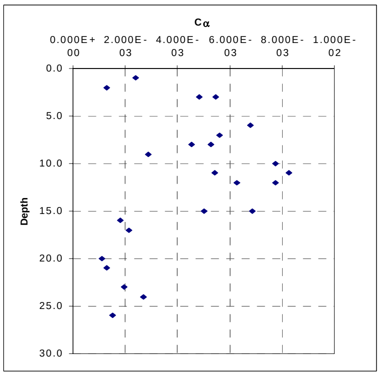

Figure 5 – Cα : Creep settlement

In order to consider the dispersion of the consolidations characteristics of the soft layer, three cases were consider:

- Base case, corresponding to the arithmetic mean for each one of the characteristics
- Worst Case, considering the worst set of characteristics
- Optimistic Case, considering the best set of characteristics

Soil Characteristics	Symbol	Unit	Opt. case	Base Case	Worst Case
Compression Index	Cc	-	0.6	0.89	1
Vertical Consolidation Coefficient	Cv	m2/s	4.00E-08	2.00E-08	1.80E-08
Ratio radial to vertical consolidation	Cr/Cv	-	3	3	3
Radial Consolidation Coefficient	Cr	m2/s	1.20E-07	6.00E-08	5.40E-08
Void Ratio	eo	-	1.8	2.148	2.32
Creep Settlement	Cα	-	0.0086	0.0095	0.0099
Creep Settlement (Cα (1+eo))	Cαe	-	0.0240	0.0300	0.0330
Clay Bulk Density	γ	kN/m3	14.6	14.6	14.6

Table 1 – Subsoil characteristics

3. ANALYSIS OF THE BASIC SOLUTION

3.1. Need for consolidation:

Under the load of the platform, the natural settlement over a period of 25 years, life time of the Power Plant, the expected natural settlement of the platform would have been in excess of 1.0 m, which was not compatible with the need to keep the platform above the 100 year flood level, but, more important, would have induced major maintenance work as well as oversizing of the piled foundation of the various structure (negative skin friction).

It was therefore decided by the client that the soil consolidation of the platform would be carried out prior to the start of the civil works.

3.2. Choice of solution

Several solutions were considered at the tender stage, namely:

- PVD + surcharge,
- Menard Vacuum,
- Cement Deep Mixing,
- Stone columns

The former one has the advantage of being well known and widely utilized in Vietnam. The only drawbacks was the limited raising speed of the surcharge (steps of 0.5 m every 3 weeks), due to the low initial shear strength of the soft clay, and the high cost of the surcharge, the sand being brought by barge from Can Tho region, some 200 km away.

The latter two solutions had the advantage of speed, but a higher cost. For the stone columns, the source of aggregates was even farther away than for sand, in Chau Doc, some 350 km away. The main drawback of both solutions was the need to determine at an early stage the pile layout of the heavy structures (Turbine pedestals, turbine hall …) to prevent any interference between the columns and the future piles. It was not compatible with the engineering production; it was further confirmed during the execution of the Civil Works, when some preliminary piles layouts were revised a few weeks before the execution.

The solution retained was finally PVD + surcharge for the zone at 2t/m² where the delivery of the platform was compatible with the fast track program and the use of Vacuum consolidation for the most critical zone and/or where the load bearing capacity was in excess of 2t/m². This paper will focus on the Vacuum consolidation works.

4. CONCEPT:

4.1. Principle:

As for the PVD + surcharge method, we pre-aged the clay (Buismann method). The various stages of the design are as follows:

a) Calculation of the consolidation phenomenon that would occur without any treatment during the considered period,
b) Considering the available preload deduction of the necessary consolidation ratio to be reached in order to catch up the primary consolidation settlement and the secondary consolidation settlement if required,
c) Calculation of the needed vertical drain mesh characteristics considering the available pre-consolidation period.

The method is illustrated below:

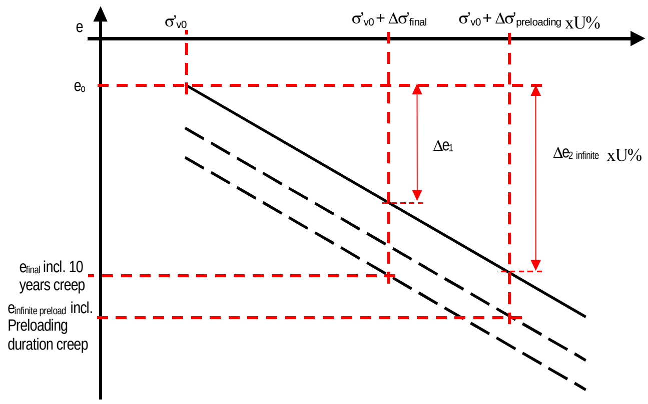

Figure 6 – Buismann method

4.2. Difference between surcharge and vacuum consolidation:

In the classical application of a surcharge, the effective stress in the soil mass is increased by increasing the total stress of the preload weight. Vacuum consolidation preloads the entire soil mass by reducing the pore pressure while maintaining an unchanged total stress.

The particularity of the Vacuum process as developed by JM. COGNON (Cognon, 1991 – Cognon and Al.) is the dewatering below the membrane, permanently keeping a gas phase between the membrane and the lowered ground water.

The P', Q' diagram (Fig. 7) allows to understand the major difference between surcharge and Vacuum.

The Vacuum process evolves from point A to point E, or below the K_0 line. The surcharge process evolves from point A to point B, or towards the K_F line.

This demonstrates that Vacuum consolidation prevents all lateral movement and even creates the opposite phenomenon of inward movement to the treated area.

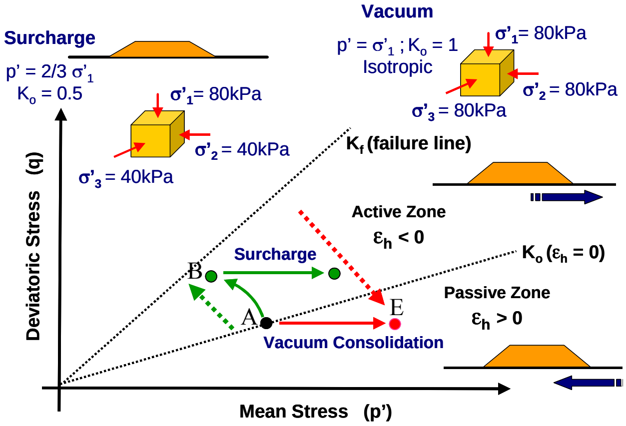

Figure 7 – Stress path for Vacuum Process

4.3. *Vacuum Consolidation installation:*

Following the installation of vertical drain and the sand blanket, tubular horizontal drain are laid, followed by an HDPE membrane. The membrane is tucked on the periphery into the soft clay to ensure imperviousness.
The horizontal drains are connected through the membrane to vacuum and dewatering pumps. This allows to create and to maintain a vacuum below the membrane and in all the soil block where vertical drains are installed.

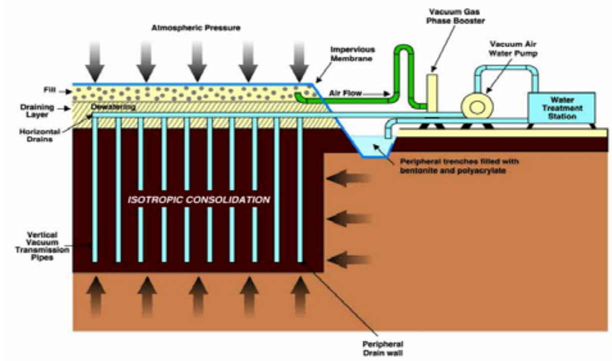

Figure 8 - Vacuum Consolidation installation

5. DESIGN FOR CA MAU PROJECT:

5.1. *Hypothesis:*

The soil conditions are described in table 1 above, considering the three cases.
4 load bearing cases were considered; 2, 5, 8 and 10 t/m².

A pre-consolidation of the clay layer to 100 % of the primary settlement under the load bearing case and an anticipation of the 10 years creep settlement is considered.

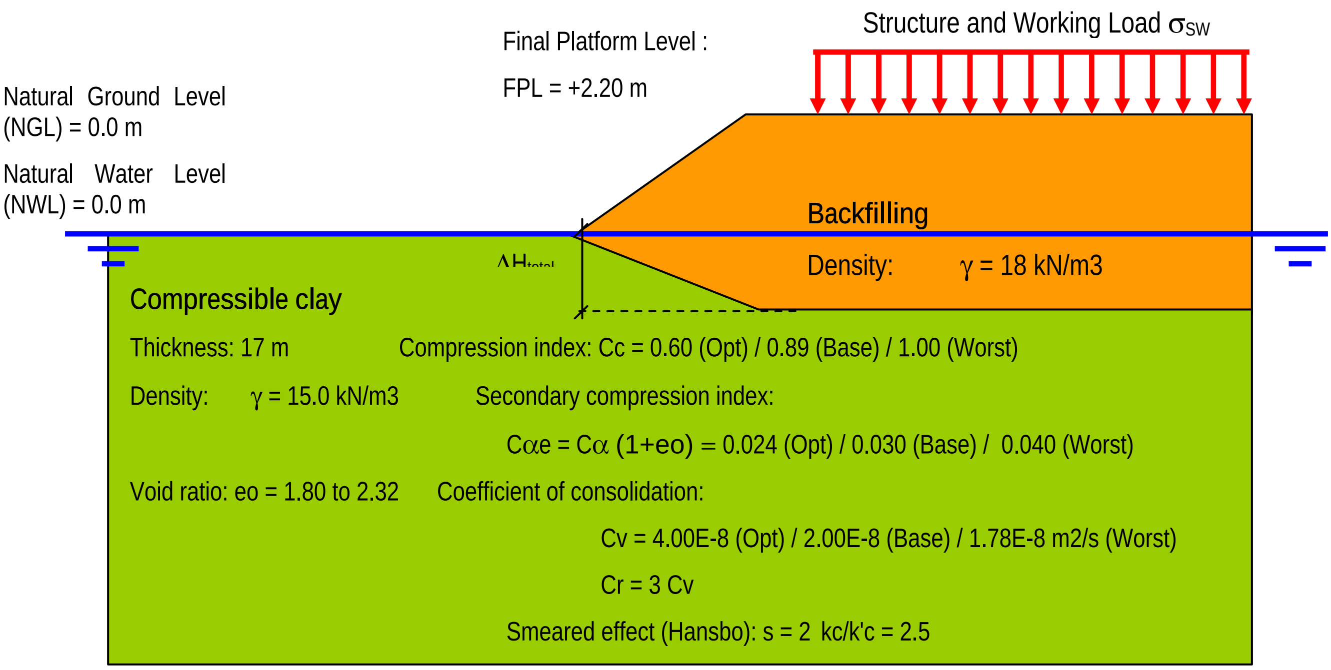

Figure 9 – Typical Cross section

5.2. *Target settlement:*

The total settlement will depend not only on the design load bearing capacity but also the load from the embankment and also the amount of fill necessary to maintain the platform at its design level.

There is therefore an iterative calculation, the final settlement depending upon the total quantity of compensation fill.

The secondary settlement being independent from the load, we determine first the amount of settlement corresponding to a 10 years creep settlement.

Then the amount of primary settlement, which is dependent upon the total load is determined.

a) Creep Settlement:
The creep settlement is only time dependent:

$$\Delta e_{s-10years} = C_{\alpha e} \log\left(\frac{T_{final}}{T_{initial}}\right) \quad (1)$$

and

$$\Delta H_{s-10years} = H * \frac{\Delta e_{s-10years}}{1+e_o} \quad (2)$$

There are several approaches to the time of appearance of the creep settlement phenomenon:

- The first approach considers that this phenomenon is starting immediately when the soft clay is loaded more than to its normally consolidated stress,
- The second approach considers that this phenomenon is starting after the primary consolidation settlement of the soft clay is finished (which, theoretically, is for an infinite time) or nearly finished,
- The third approach consists in considering that the primary and secondary consolidation phenomenon are closely linked to each other and occur at the same time.

Indeed, the primary consolidation being dependant of the additional pore pressure decrease inside the clay layer, this effect depends on the depth at which the clay is considered: near a drainage layer, i.e. at the top and at the bottom of the clay layer when the clay is drained on both sides or near a vertical drain, the primary consolidation speed is very high and thus the secondary (creep) settlement begins very quickly whereas at the middle of the clay layer and far from the vertical drains, the primary consolidation speed is lower and the secondary (creep) settlement begins later.

We prefer to remain on the safe side and thus consider that the creep settlement starts at day 1. Working (1) and (2) for the various design load bearing case, we have:

Secondary Consolidation (10 years)		
Optimistic case	Base Case	Worst Case
0.52 m	0.58 m	0.60 m

Table 2: Creep settlement over 10 years

b) Primary Consolidation:
The primary consolidation is dependent upon the increase of stress at any location:

$$\Delta e_{primary} = Cc * \log\left(\frac{\sigma'_o + \Delta\sigma_{final}'}{\sigma'_o}\right) \quad (3)$$

With:

$$\Delta\sigma'_{final} = \Delta H_{tot}\left(\gamma_{fill} - \gamma_{water}\right) + \left(FPL - NGL\right) * \gamma_{fill} + \sigma_{SW}$$
(4)

σ_{sw}: Design load bearing

In the initial situation, the stress at the middle of the clay layer is:

$$\sigma'_o = \frac{17}{2}\left(14.6 - 10\right) = 39.1 \text{ kPa}$$

and the initial void ratio is $e_0 = 2.148$.

Working (3), (4) and (2), we finally have:

Primary Consolidation (10 years)			
Load case	*Optim. case*	*Base Case*	*Worst Case*
2 t/m²	1.75 m	2.42 m	2.61 m
5 t/m²	2.15 m	2.93 m	3.15 m
8 t/m²	2.45 m	3.33 m	3.57 m
10 t/m²	2.62 m	3.53 m	3.79 m

Table 3: Primary Settlement

And finally the final expected settlement is as follow:

Target Settlement (10 years)			
Load case	*Optim. case*	*Base Case*	*Worst Case*
2 t/m²	2.27 m	3.00 m	3.21 m
5 t/m²	2.66 m	3.51 m	3.75 m
8 t/m²	2.97 m	3.91 m	4.17 m
10 t/m²	3.14 m	4.11 m	4.39 m

Table 4: Total target settlement

It is to be noted that the target settlement is not to be considered as an absolute value in the determination of the end of the consolidation. It gives a good indication on the total level of fill compensation to consider but the end of the consolidation is determined by back analysis and actual consolidation process (See chapter 6 Monitoring thereafter).
Those values are used mostly in the determination of a targeted consolidation ratio, under a higher temporary load, the soil parameters being the same, as follows in the next step of the calculation.
The stress corresponding to the above targeted settlement is as given in following table:

Stress at mid layer				
Load Case	*2 t/m²*	*5 t/m²*	*8 t/m²*	*10 t/m²*
Initial stress (σ'_o)	39 kPa	39 kPa	39 kPa	39 kPa
Additional Stress ($\Delta\sigma'$)	86 kPa	120 kPa	154 kPa	175 kPa
Total Stress	125 kPA	159 kPa	193 kPa	214 kPa
Δe_1	0.448	0.542	0.616	0.658

Table 5: Final Stress at mid-layer / Target void ratio reduction

5.3. Target consolidation ratio:

At the end of the pre-consolidation period, the stress depends on the applied loading made of Vacuum void pressure and additional backfilling if any.

In our cases, the foreseen Vacuum pressure is 70 kPa and the following preloading thickness is foreseen:

- Nothing for the 2t/m² case, resulting in a total stress at the middle of the clay layer of 39 + (86-20) + 70 = 175 kPa,
- 2 m for the 5t/m² case, resulting in a total stress at the middle of the clay layer of 39 + (120-50) + 70 + 2*18 = 215 kPa,

- 5 m for the 8t/m² case, resulting in a total stress at the middle of the clay layer of 39 + (154-80) + 70 + 5*18 = 273 kPa,
- 7 m for the 10t/m² case, resulting in a total stress at the middle of the clay layer of 39 + (175-100) + 70 + 7*18 = 310 kPa

Working formula (3) for the various cases for the stress after construction and under the preconsolidation load, we determine the corresponding variation in void ratio Δe_{2inf}.

On the other hand, one shall consider the effect of the creep settlement on the void ratio $\Delta e_{s\,(8\ months \Rightarrow 10\ years)}$, taken between the end of the preconsolidation (taken as 8 months) and 10 years.

The aim of the pre consolidation is to have:

$U\% * \Delta e_{2\,inf} \geq \Delta e_1 + \Delta e_{s\,(8\ months \Rightarrow 10\ years)}$

ie $$U\% \geq \frac{\left(\Delta e_1 + \Delta e_{s(8 months => 10 years)}\right)}{\Delta e_{2\,\inf}}$$

and finally:

Target Consolidation ratio at end of pre-consolidation			
Load case	*Optim. case*	*Base Case*	*Worst Case*
2 t/m²	83 %	84 %	84 %
5 t/m²	87 %	87 %	87 %
8 t/m²	87 %	87 %	87 %
10 t/m²	87 %	87 %	87 %

Table 5: Target consolidation ratio during preloading

5.4. Theoretical consolidation ratio:

Using Carillo's formulas modified by Hansbo in order to take into account the smeared zone, the consolidation ratio is calculated as follows:

$$U_v \cong \left(\frac{1}{1 + \frac{1}{2T_V{}^3}} \right)^{1/6} \quad \text{with} \quad T_V = \frac{C_V t}{H^2}$$

$$U_r = 1 - \exp\left(\frac{-8C_r t}{D^2 \mu} \right) \quad \text{with}$$

$$\mu = \ln\left(\frac{n}{s}\right) - \frac{3}{4} + \frac{k_h}{k'_h}\ln(s)$$

n = D / d

D = 1.13 L (square network where L is the distance between 2 drains)

s = 2 (ratio between diameter of smeared zone and equivalent diameter of drain)

kh / k'h = 2.5 (ratio between horizontal permeability in the smeared zone and in a normal zone)

Finally, Carillo gives:

(1-U) = (1-Ur)(1-Uv)

With vertical drains of 100 mm * 3 mm, having an equivalent diameter of 6.5 cm, arranged on a regular square grid of 0.9 * 0.9 m2 and with a consolidation period of 5.5 months, we finally get a consolidation ratio of 90%, higher than the necessary consolidation ratio for any and all load cases.

For the worst case, the consolidation is 87 %, sufficient to fulfill the target consolidation ratio.

5.5. Residual settlement:

At the end of the pre-consolidation and removal of the surcharge, creep settlement will continue, but on the basis of a minimum "10 years old" pre-aged clay (see above) and the theoretical value of such remaining creep settlement will be 10 years after the end of the treatment:

$$\Delta e = \frac{C_\alpha}{1 + e_0} * \log\left(\frac{T_{initial} + 10 years}{T_{initial}} \right)$$

Where $T_{initial}$ > 10 years

Thus $\Delta e < \frac{C_\alpha}{1 + e_0} * \log(2) = 9E-3$, and finally ΔH < 48 mm, well below the required 100 mm.

6. MONITORING:

As mentioned, the reaching or not of the target settlement is not a good indicator of the state of consolidation achieved in the clay during the Pre-consolidation, as the target settlement is highly dependent upon the representativity of the soil samples and the quality of the laboratory tests.
The evolution of the consolidation is however more explicit and the follow up of this evolution of the consolidation process is more adequate and closely monitor.
Such evolution was then evaluated using two methods:

- Asaoka Method and
- Pore pressure decrease method

6.1. Instrumentation:

The monitoring system consisted in:

- Settlement Plates, which aim was to monitor the settlement at the surface of the site,
- Multidepth settlement gages, which aim was to monitor the settlement of the soil at different depths,
- Pore Pressure sensors, which aim was to monitor the pore pressure decrease during consolidation,
- Vacuum gages, which aim was to monitor the Vacuum pressure below the membrane in the soil,
- Inclinometers, which aim was to check the stability of the embankment.

Type	Aim	Density (1 unit for)	Frequency of Readings	Analysis	
				Type	Frequency
Settlement plates	Monitoring of Settlement	25 * 25 m2	1 per week	Asaoka	1 per month
Vacuum Gauges	Monitoring of the applied depression	80 * 80 m2	1 per week	Check vacuum	1 per week
Pore Pressure sensor	Monitoring of pore pressure decreased speed	150 * 150 m2	1 per week	Pore pressure decrease speed	1 per month
Multidepth Settlement	Monitoring of settlement at various depth of the clay layer	80 * 80 m2	1 per week	Asaoka	1 per month
Inclinometers	Checking of slope stability	150 m of high slope	1 per week	Check stability	1 per week

Table 6: Instrumentation

6.2. Asaoka analysis:

a) Principle of the method:

The Asaoka analysis uses the observed values of settlement in order to estimate the consolidation parameters and thus re-check the design with actual observed data.
It consists in plotting the curve (settlement at time t_{n+1} versus settlement at time t_n), the difference $t_{n+1} - t_n$ being constant (defined as Δt).

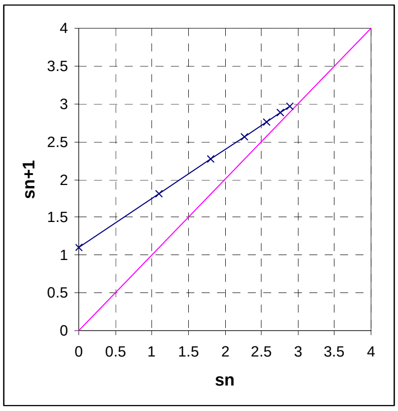

Figure 10- ASAOKA analysis

According to the Consolidation theory, for a constant loading, this curve is a straight line which characteristics allows one to estimate the various actual consolidation parameters, including the total primary consolidation settlement (which is at the cross of this line with the bisecting line).

Indeed, the theoretical settlement law is the following:

$$s(t) = s_\infty \left(1 - \frac{8}{\pi^2} \exp(-C * t) \right)$$

Where: $C = \dfrac{8C_r}{D^2 F(n)} + \dfrac{\pi^2 C_v}{4H^2}$ and

Cr is the radial consolidation ratio
Cv is the vertical consolidation ratio
t is the time
D is the diameter of influence of the vertical drains (D = 1.13 L for a square mesh, L being the mesh value)
d is the equivalent diameter of the vertical drains
n = D / d

$$F(n) = \frac{n^2}{n^2 - 1} \ln(n) - \frac{3n^2 - 1}{4n^2}$$

For a fixed Δt, we thus have:

$$s(t + \Delta t) = s_\infty \left(1 - \frac{8}{\pi^2} \exp(-C * (t + \Delta t)) \right) = s_\infty (1 - \exp(-C * \Delta t)) + \exp(-C * \Delta t) * s(t)$$

i.e. $s(t + \Delta t) = \beta_1 s(t) + \beta_0$

The above described curve is thus a straight line which slope, β_1, is related to C coefficient. The estimation of the slope of this curve thus allows to estimate this consolidation parameter. Moreover, the settlement for t infinite can also be estimated by simply reading the settlement obtained at the crossing of the curve with the bisecting line.

b) Results on Zone 1:

Figure 11 below is an example of the Asaoka analysis on the basis of the settlement readings from a plate of zone 1 (5T/m²).

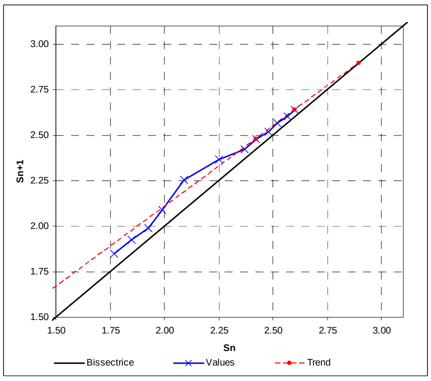

Figure 11- Example of obtained ASAOKA curve

The non-linear part of the graph corresponds to the time the surcharge is increased (embankment built-up), while the linear part starts once the full embankment height has been reached.
On the example above, the theoretical infinite primary settlement is 2.89m, while the actual settlement before unloading was 2.64m, resulting in a consolidation ratio of U = 91%, well over the required criterion of $U_{\%}$ = 87%.
The same analysis is done on every settlement plates and multidepth settlement sensors, in order to check that the obtained consolidation ratio before unloading is higher that the required value at all locations.

6.3. Pore pressure decrease analysis:

a) Principle of the method:
The pore pressure decrease methodology uses the observed values of the pore pressure decrease speed in order to estimate the consolidation parameters and thus re-check the design with actual observed datas.
The theoretical Pore pressure law is the following:

$$u(r, Cr, t) = A(r) \exp(-B(Cr)t)$$

Where:
r is the distance of the concerned point to the closest vertical drain
Cr is the radial consolidation ratio
t is the time
D is the diameter of influence of the vertical drains (D = 1.13 L for a square mesh, L being the mesh value)
d is the equivalent diameter of the vertical drains
n = D / d

$$A(r) = \frac{4uo}{D^2 F(n)} \left[\left(\frac{D}{2} \right)^2 \ln\left(\frac{r}{d/2} \right) + \frac{(d/2)^2 - r^2}{2} \right]$$

$$B(Cr) = \frac{8Cr}{D^2 F(n)}$$

$$F(n) = \frac{n^2}{n^2 - 1} \ln(n) - \frac{3n^2 - 1}{4n^2}$$

Deriving this pore pressure law against time, for a constant r (i.e. a constant location of pore pressure sensor, which is the case) and for a constant Cr, we obtain:

$$\frac{\partial u}{\partial t} = -B(Cr)A(r)\exp(-B(Cr)t)$$ Value of the pore pressure decrease speed

For 2 different time values, T1 and T2, and for a constant r and Cr, we finally get:

$$\frac{\left.\frac{\partial u}{\partial t}\right|_{T1}}{\left.\frac{\partial u}{\partial t}\right|_{T2}} = \exp(-B(Cr)(T1-T2))$$

The coefficient B(Cr) being fixed against time and being only dependant of the actual geometry of the drain project and of Cr soil value, it is thus possible to re-estimate very simply the Cr value out of pore pressure decrease observation (of course for the same sensor as it depends on the location of the sensor against the location of the drain) by calculating the ratio of pore pressure decrease speeds at 2 different time values.

b) Results on Zone 1:

Figure 12 below is an example of the pore pressure decrease speed analysis done on the readings from a pore pressure sensor in zone 1 (5T/m²):

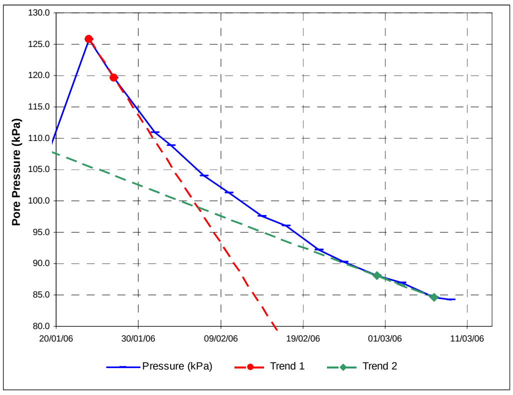

Figure 12- Example of obtained Pore Pressure curve

The pore water pressure increases rapidly, together with the raising of the embankment and reach its highest level at the end of the full embankment height, then decreases rapidly. Two relevant trend lines are chosen, the ratio between the slopes of these two lines leading to the recalculated value of Cr, governing factor of the consolidation speed. On the above example, the obtained Cr value is 1.2E-7 m²/s.

The values obtained by this method being much more relevant than correlation from soil investigations analysis, one can re-run multi-step settlement and soil effective stress analysis and then consolidation ratio calculation.

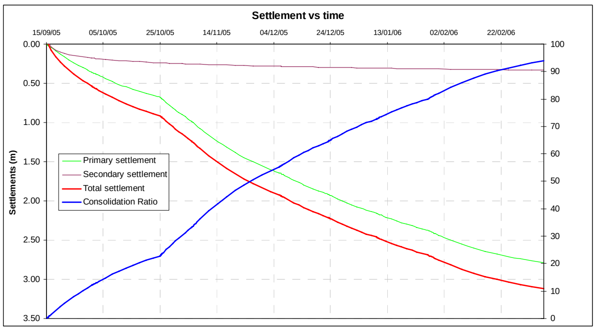

Figure 13- Settlement and Consolidation Ratio Curve

7. CONCLUSION

After Thailand, Korea and Malaysia, Vietnam can now be added to the list of countries in Asia where Vacuum Consolidation has been applied with success.
This solution is an ideal alternative to the Vertical Drain and surcharge method for tight schedule project, in particular for very soft soil where stability of the embankment can reduce drastically the speed of embankment raising.
With vast areas of soft soil deposit close to urban centres in need of space for their development, dwindling resource of material, forcing higher the cost of surcharge, the Vacuum consolidation particularly suitable for Vietnam.

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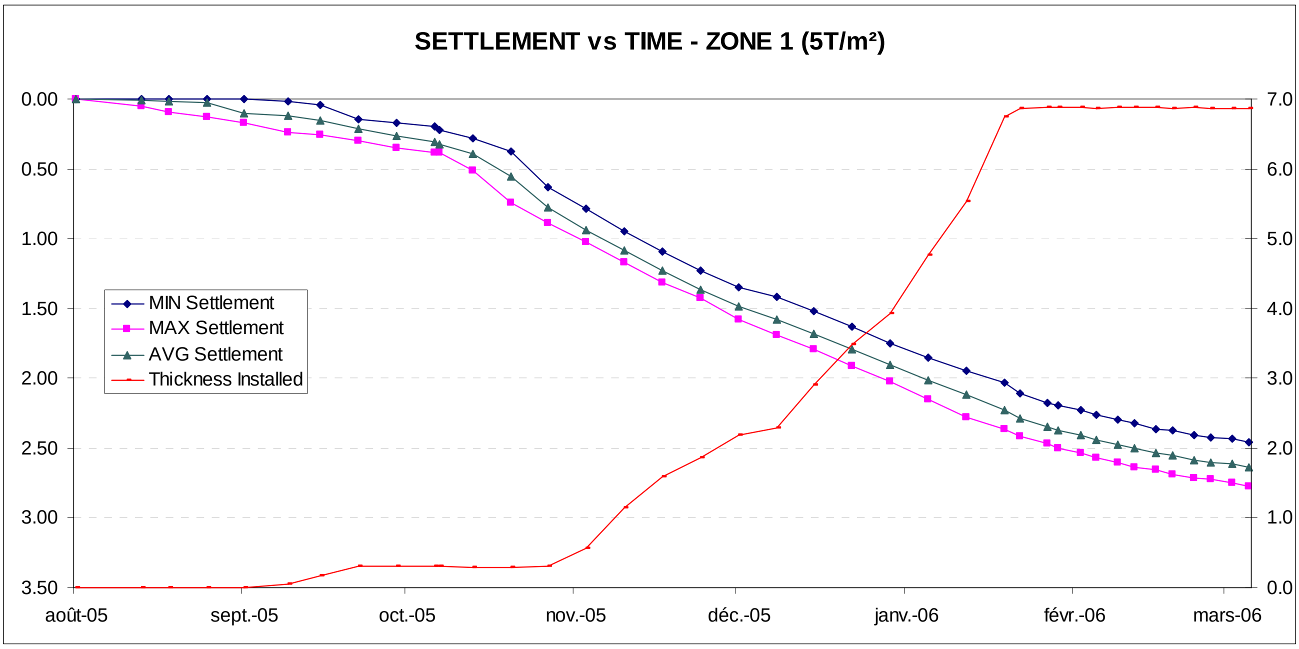

Figure 14 – Settlement and fill record